Chapter 5
System Stability

5-1. Modes of Failure

The loads exerted on wall/soil system tend to produce a variety of potential failure modes. These failure modes, the evaluation of the loads on the system, and selection of certain system parameters to prevent failure are discussed in this chapter.

a. Deep-seated failure. A potential rotational failure of an entire soil mass containing an anchored or cantilever wall is illustrated in Figure 5-1. This potential failure is independent of the structural characteristics of the wall and/or anchor. The adequacy of the system (i.e. factor of safety) against this mode of failure should be assessed by the geotechnical engineer through conventional analyses for slope stability (EM 1110-2-1902). This type of failure cannot be remedied by increasing the depth of penetration nor by repositioning the anchor. The only recourse when this type of failure is anticipated is to change the geometry of retained material or improve the soil strengths.

b. Rotational failure due to inadequate pile penetration. Lateral soil and/or water pressures exerted on the wall tend to cause rigid body rotation of a cantilever or anchored wall as illustrated in Figure 5-2. This type of failure is prevented by adequate penetration of the piling in a cantilever wall or by a proper combination of penetration and anchor position for an anchored wall.

c. Other failure modes. Failure of the system may be initiated by overstressing of the sheet piling and/or anchor components as illustrated in Figures 5-3 and 5-4. Design of the anchorage to preclude the failure depicted in Figure 5-4a is discussed later in this chapter. Design of the structural components of the system is discussed in Chapter 6.

5-2. Design for Rotational Stability

a. Assumptions. Rotational stability of a cantilever wall is governed by the depth of penetration of the piling or by a combination of penetration and anchor position for an anchored wall. Because of the complexity of behavior of the wall/soil system, a number of simplifying assumptions are employed in the classical design techniques. Foremost of these assumptions is that the deformations of the system are sufficient to produce limiting active and passive earth pressures at any point on the wall/soil interface. In the design of the anchored wall, the anchor is assumed to prevent any lateral motion at the anchor elevation. Other assumptions are discussed in the following paragraphs.

b. Preliminary data. The following preliminary information must be established before design of the system can commence.

(1) Elevation at the top of the sheet piling.

(2) The ground surface profile extending to a minimum distance of 10 times the exposed height of the wall on either side.

(3) The soil profile on each side of the wall including location and slope of subsurface layer boundaries, strength parameters (angle of internal friction ϕ, cohesive strength c, angle of wall friction δ, and wall/soil adhesion) and unit weight for each layer to a depth below the dredge line not less than five times the exposed height of the wall on each side.

(4) Water elevation on each side of the wall and seepage characteristics.

(5) Magnitudes and locations of surface surcharge loads.

(6) Magnitudes and locations of external loads applied directly to the wall.

c. Load cases. The loads applied to a wall fluctuate during its service life. Consequently, several loading conditions must be defined within the context of the primary function of the wall. As a minimum, a cooperative effort among structural, geotechnical, and hydraulic engineers should identify the load cases outlined to be considered in the design.

(1) Usual conditions. The loads associated with this condition are those most frequently experienced by the system in performing its primary function throughout its service life. The loads may be of a long-term sustained nature or of an intermittent, but repetitive, nature. The fundamental design of the system should be optimized for these loads. Conservative factors of safety should be employed for this condition.

(2) Unusual conditions. Construction and/or maintenance operations may produce loads of infrequent occurrence and are short duration which exceed those of the usual condition. Wherever possible, the sequence of operations should be specified to limit the magnitudes

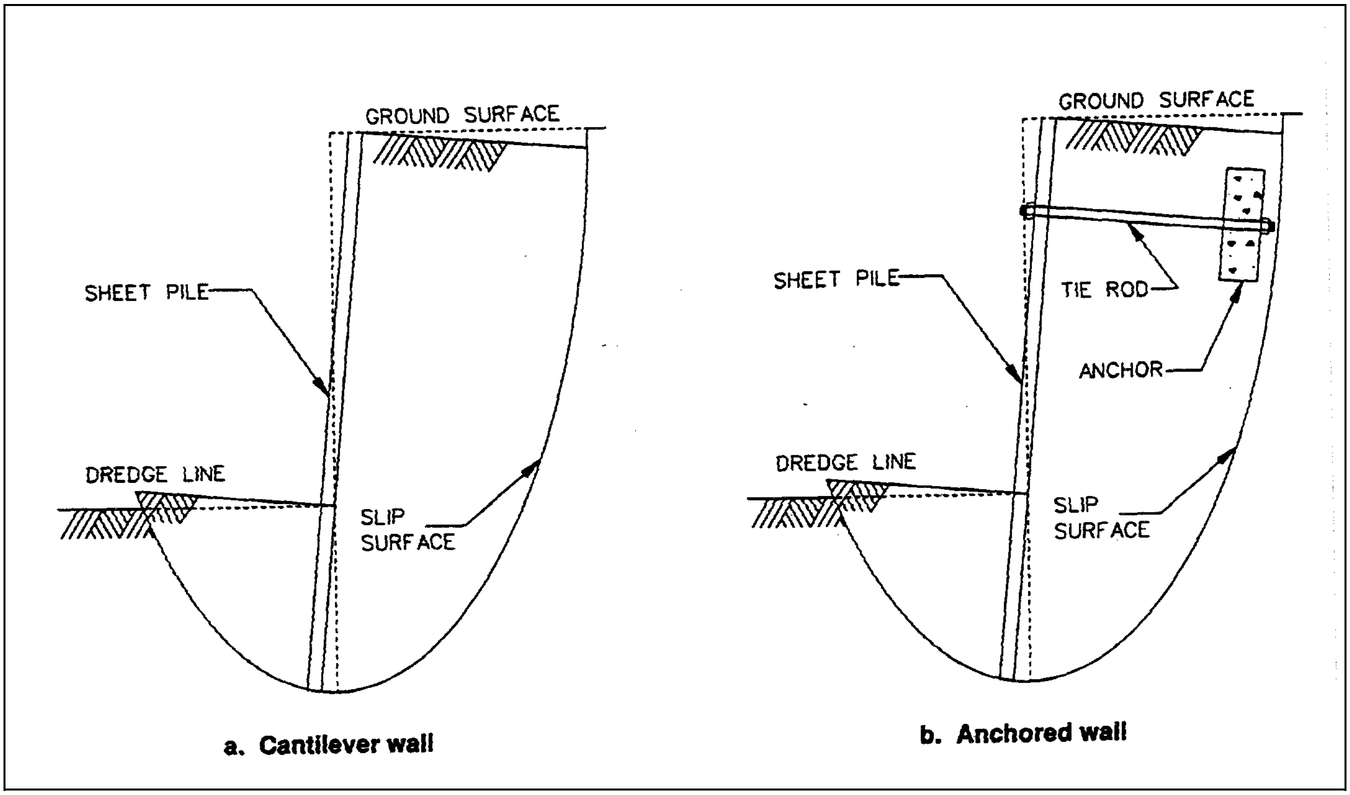

Figure 5-1. Deep-seated failure

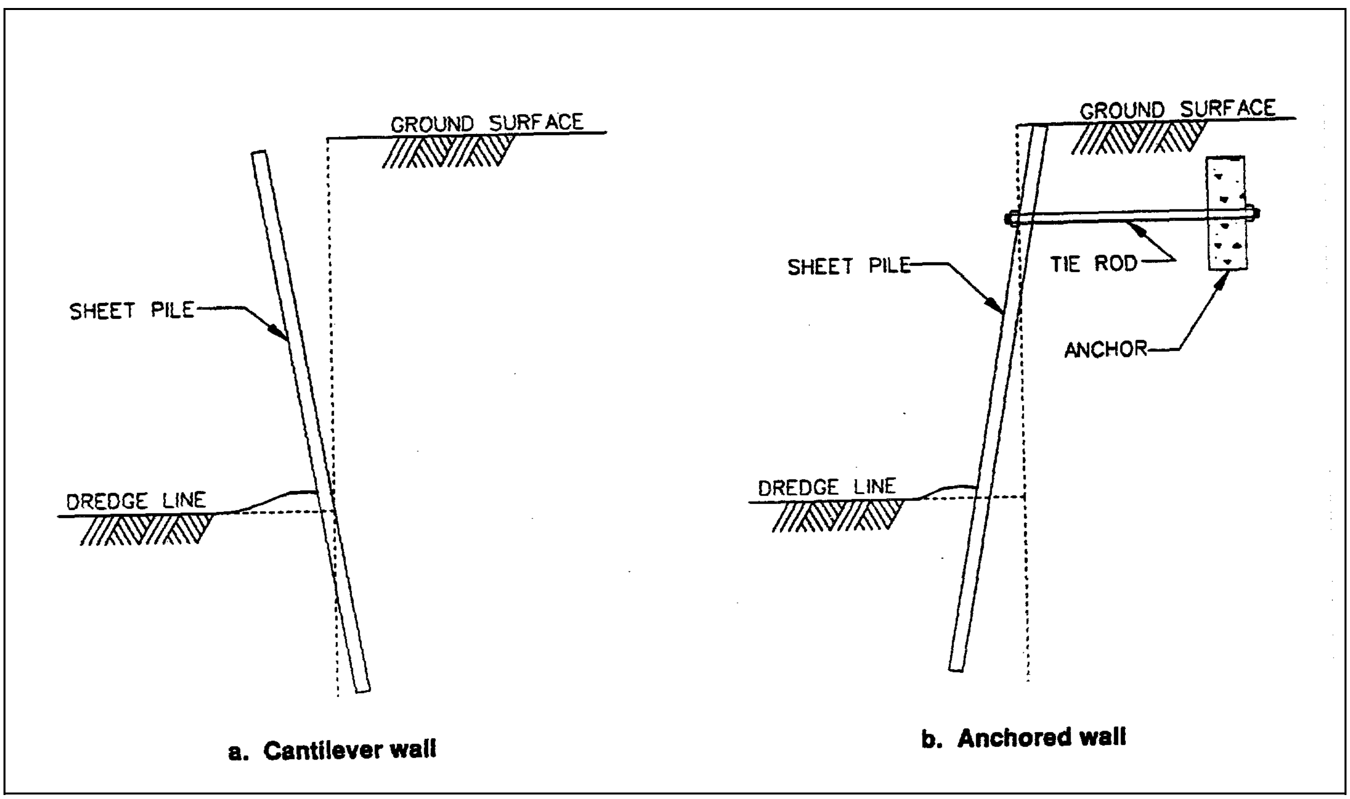

Figure 5-2. Rotational failure due to inadequate penetration

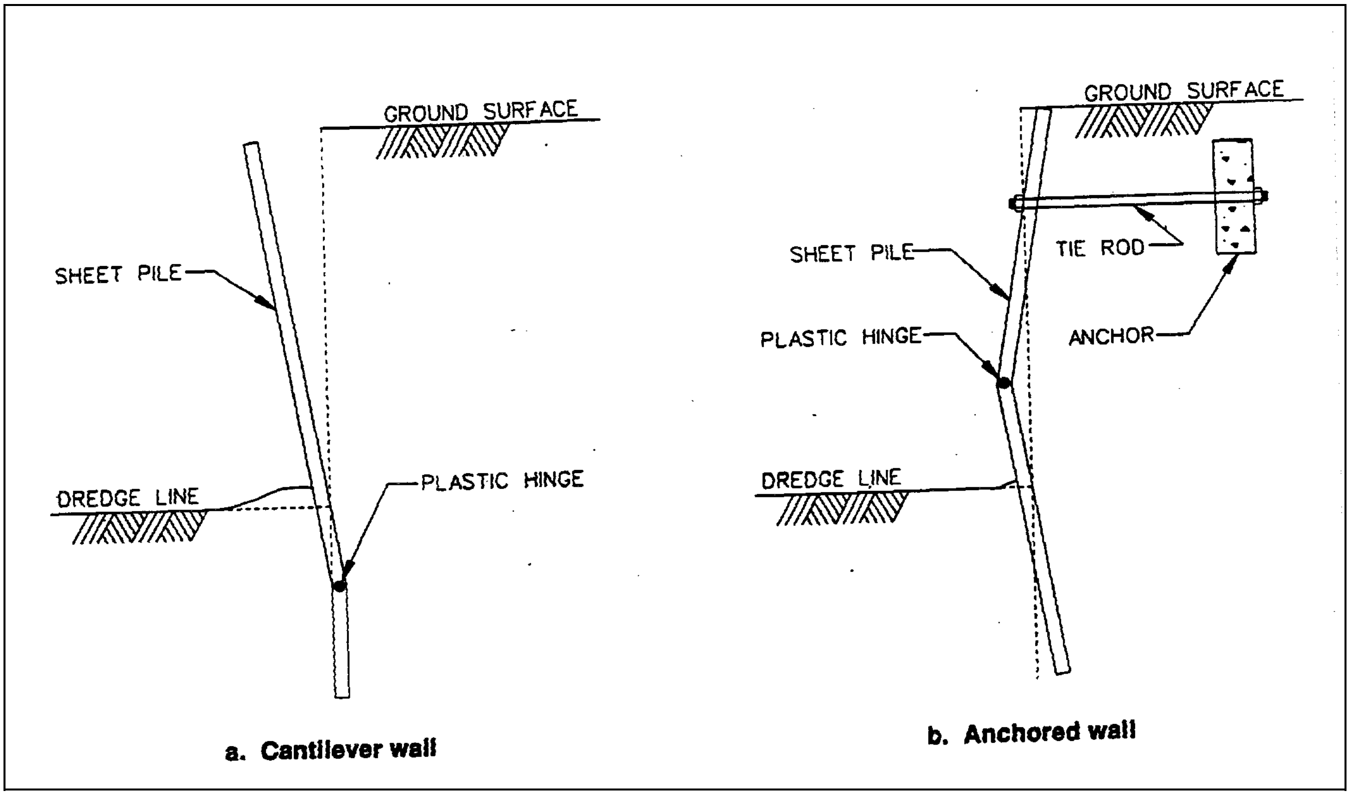

Figure 5-3. Flexural failure of sheet piling

and duration of loading, and the performance of the wall should be carefully monitored to prevent permanent damage. Lower factors of safety or higher material stresses may be used for these conditions with the intent that the system should experience no more than cosmetic damage.

(3) Extreme conditions. A worst-case scenario representing the widest deviation from the usual loading condition should be used to assess the loads for this case. The design should allow the system to sustain these loads without experiencing catastrophic collapse but with the acceptance of possible major damage which requires rehabilitation or replacement. To contrast usual and extreme conditions, the effects of a hurricane on a hurricane protection wall would be the "usual" condition governing the design, while the loads of the same hurricane on an embankment retaining wall would be "extreme."

d. Factors of safety for stability. A variety of methods for introducing "factors of safety" into the design process have been proposed; however, no universal procedure has emerged. In general, the design should contain a degree of conservatism consistent with the experience of the designer and the reliability of the values assigned to the various system parameters. A procedure which has gained acceptance in the Corps of Engineers is to apply a factor of safety (strength reduction factor) to the soil strength parameters ϕ and c while using "best estimates" for other quantities. Because passive pressures calculated by the procedures described in Chapter 4 are less likely to be fully developed than active pressures on the retaining side, the current practice is to evaluate passive pressures using "effective" values of ϕ and c given by

$$\tan(\phi_{eff}) = \tan(\phi) \;/\; FSP \tag{5-1}$$

and

$$c_{eff} = c \;/\; FSP \tag{5-2}$$

where

FSP = factor of safety for passive pressures

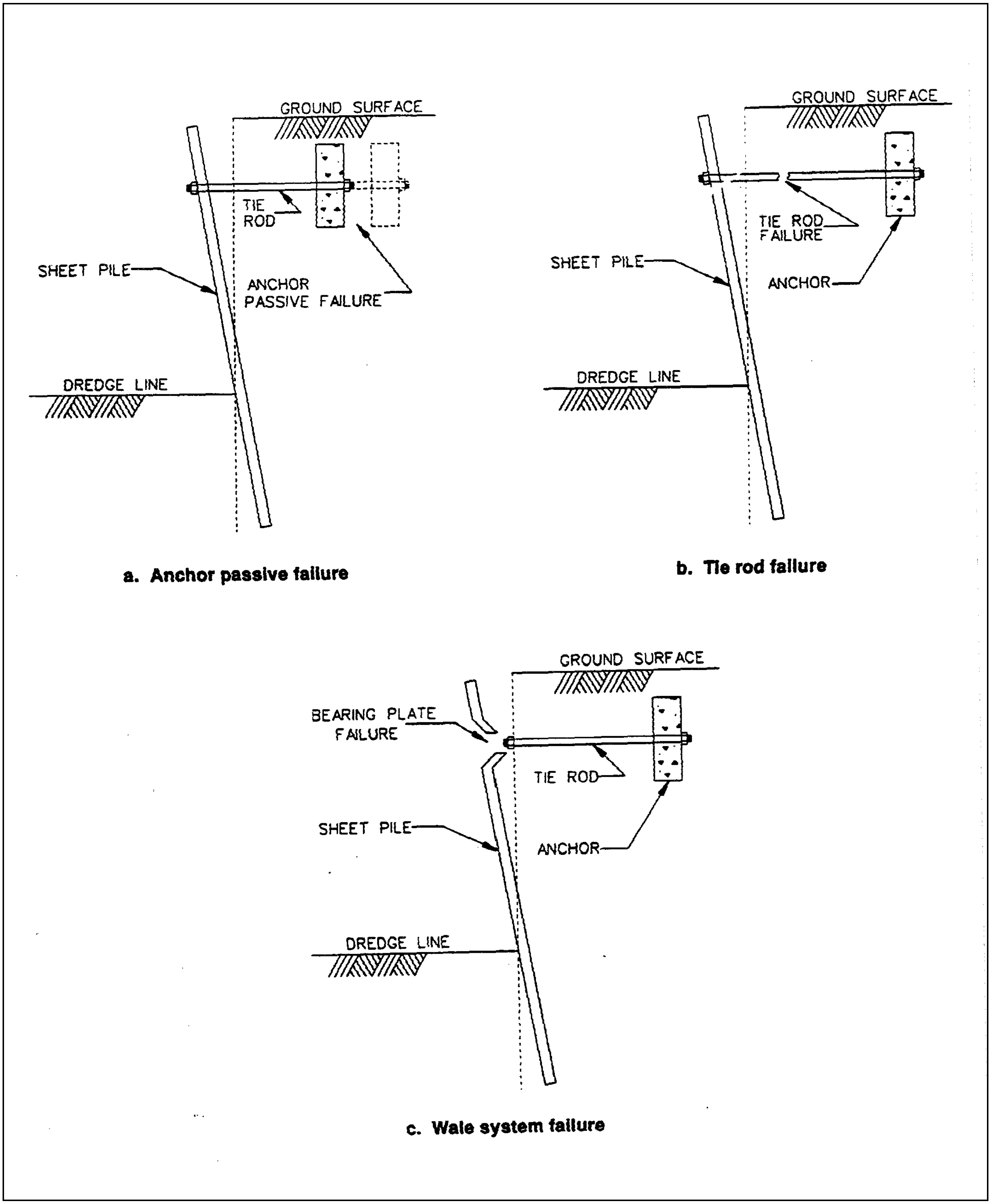

Figure 5-4. Anchorage failures

Minimum recommended values of FSP are given in Table 5-1. A factor of safety FSA may be applied for active pressures, however it is considered sufficient to use an FSA = 1 in most cases unless deformations of the wall are restricted.

Table 5-1
Minimum Safety Factors for Determining the Depth of Penetration Applied to the Passive Pressures

Loading Case	Fine-Grain Soils	Free-Draining Soils
Floodwalls		
Usual	1.50 Q-Case 1.10 S-Case	1.50 S-Case
Unusual	1.25 Q-Case 1.10 S-Case	1.25 S-Case
Extreme	1.10 Q-Case 1.10 S-Case	1.10 S-Case
Retaining Walls		
Usual	2.00 Q-Case 1.50 S-Case	1.50 S-Case
Unusual	1.75 Q-Case 1.25 S-Case	1.25 S-Case
Extreme	1.50 Q-Case 1.10 S-Case	1.10 S-Case

e. Net pressure distributions. Evaluations of the pressures by the processes described in Chapter 4 result in a number of pressure distributions.

(1) Active soil pressures due to retained side soil.

(2) Passive soil pressures due to retained side soil.

(3) Pressures due to surcharge loads on retained side surface. (Effects of surcharge loads are included in the soil pressures when a wedge method is used.)

(4) Active soil pressures due to dredge side soil.

(5) Passive soil pressures due to dredge side soil.

(6) Pressures due to surcharge loads on dredge side surface.

(7) Net water pressures due to differential head.

For convenience in calculations for stability, the individual distributions are combined into "net" pressure distributions according to:

"NET ACTIVE" PRESSURE = retained side active soil pressure
- dredge side passive soil pressure
+ net water pressure
(+ pressure due to retained side surcharge)
(- pressure due to dredge side surcharge)

"NET PASSIVE" PRESSURE = retained side passive soil pressure
- dredge side active soil pressure
+ net water pressure
(+ pressure due to retained side surcharge)
(- pressure due to dredge side surcharge)

In these definitions of net pressure distributions, positive pressures tend to move the wall toward the dredge side. Typical net pressure diagrams are illustrated in Figure 5-5.

f. Stability design for cantilever walls. It is assumed that a cantilever wall rotates as a rigid body about some point in its embedded length as illustrated in Figure 5-2a. This assumption implies that the wall is subjected to the net active pressure distribution from the top of the wall down to a point (subsequently called the "transition point") near the point of zero displacement. The design pressure distribution is then assumed to vary linearly from the net active pressure at the transition point to the full net passive pressure at the bottom of the wall. The design pressure distribution is illustrated in Figure 5-6. Equilibrium of the wall requires that the sum of horizontal forces and the sum of moments about any point must both be equal to zero. The two equilibrium equations may be solved for the location of the transition point (i.e. the distance z in Figure 5-6) and the required depth of penetration (distance d in Figure 5-6). Because the simultaneous equations are nonlinear in z and d, a trial and error solution is required.

g. Stability design for anchored walls. Several methods for anchored wall design have been proposed

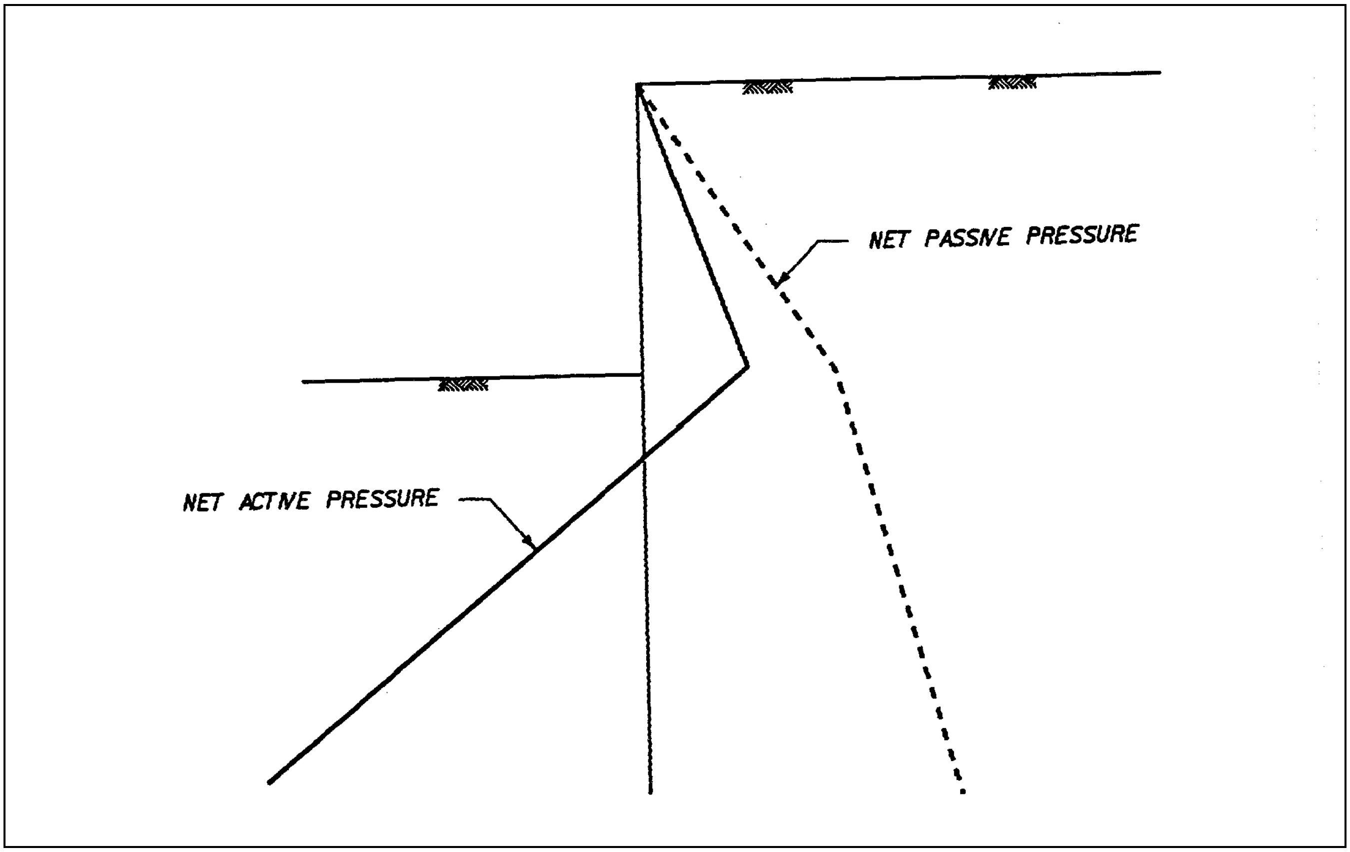

Figure 5-5. Typical net pressure distributions

and classified as the "Free Earth" method (implied in Figure 5-2b) and variations of the "Fixed Earth" hypothesis. Research and experience over the years have shown that walls designed by the Free Earth method are sufficently stable walls with less penetration than those designed by the Fixed Earth method. Because of the flexibility of the sheet piling, the Free Earth method predicts larger moments than those that actually occur. This shortcoming of the Free Earth method is overcome by using Rowe's moment reduction curves, as described in Chapter 6. In the Free Earth method, the anchor is assumed to be a rigid simple support about which the wall rotates as a rigid body as shown in Figure 5-2b. Despite the tendency of the wall to produce a passive condition in the retained soil above the anchor, it is assumed that the wall is only subjected to the net active pressure distribution as illustrated in Figure 5-7. The required depth of penetration (*d* in Figure 5-7) is determined from the equilibrium requirement that the sum of moments about the anchor must be zero. After the depth of penetration has been determined, the anchor force is obtained from equilibrium of horizontal forces. Because the position of the anchor affects both depth of penetration and anchor force, it will be necessary to consider several anchor positions to arrive at the optimal combination. For an initial estimate, the anchor may be assumed to lie at a distance below the top of the wall equal to one-fourth to one-third of the exposed wall height.

h. Anchor design. The anchor force calculated in the stability analysis was obtained from equilibrium of a typical 1-foot slice of the wall. In the actual system the anchor support is provided by discrete tie rods attached to the wall through wales and to another support mechanism (termed the "anchor" herein) at their ends and remote from the wall. Structural design of the tie rods and wales is discussed in Chapter 6. A variety of anchor configurations are illustrated in Figure 2-2. Capacities of some anchor configurations are discussed in the following paragraphs. The soil strength parameters appearing in the equations associated with anchor design should be consistent with the properties (S-case or Q-case) used for stability design. In all cases the capacity of the anchor should be sufficient to develop the yield strength of the tie rods (Chapter 6).

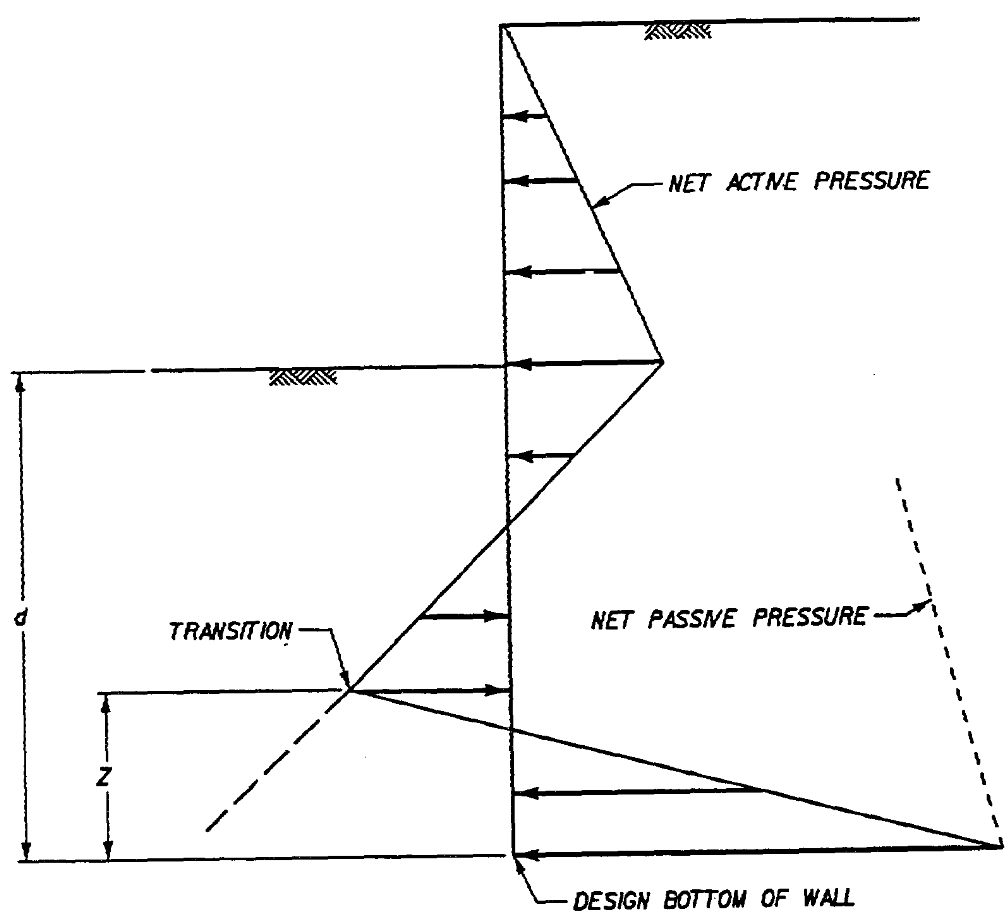

Figure 5-6. Design pressure distribution for cantilever wall

(1) Continuous anchors. A continuous anchor consists of a sheet pile or concrete wall installed parallel to the retaining wall as illustrated in Figures 2-2a and 2-2b. The continuous anchor derives its resistance from differential passive and active pressures produced by interaction with the surrounding soil.

(a) Anchor location. The minimum distance from the retaining wall at which an anchor wall must be placed to develop its full capacity is illustrated in Figure 5-8 for a homogeneous soil system. Under the assumptions employed in the stability analysis of the retaining wall, a zone of soil (bounded by line ab in Figure 5-8) behind the retaining wall is at its limiting active state. To permit development of passive pressures, an additional zone of soil (bounded by line bc in Figure 5-8) must be available. In addition, if the anchor wall intersects the line ac in Figure 5-8, interaction between the anchor wall and the retaining wall may increase the soil pressures on the retaining wall, thus invalidating the previous stability analysis. For nonhomogeneous soil systems, the boundaries defining minimum spacing of the anchor wall may be estimated by the procedures used in the "Fixed Surface" wedge method described in CWALSHT User's Guide (USAEWES 1990).

(b) Full anchor capacity. Active and passive pressures developed on the anchor wall are shown in Figure 5-9 for a homogeneous soil system where *h/H* is 1/3 to 1/2 (Teng (1962) and Terzaghi (1943)). The capacity of the anchor wall is given by

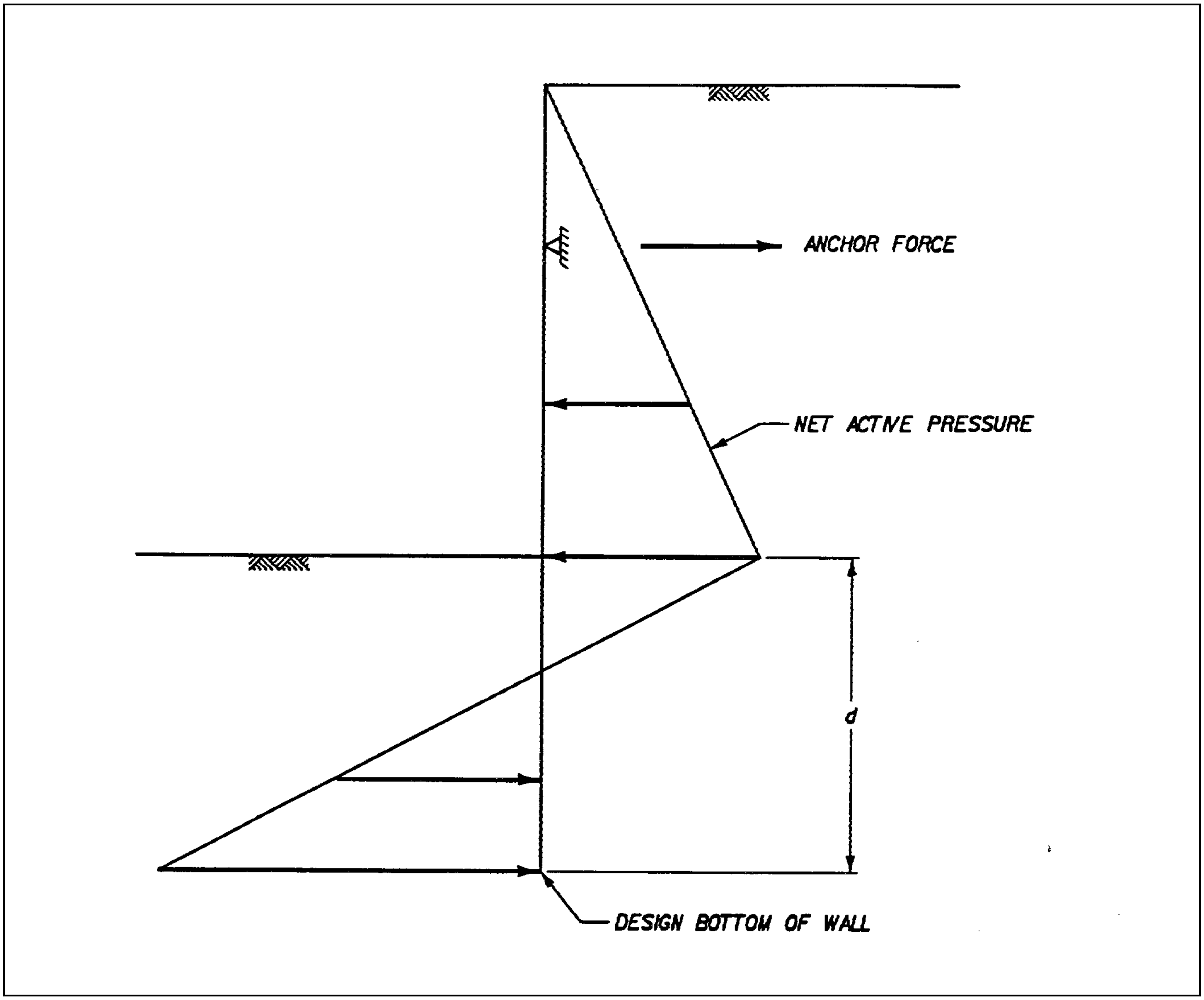

Figure 5-7. Design pressure distribution for free earth design of anchored walls

$$C_a = P_P - P_A \tag{5-3}$$

where

C_a = anchor wall capacity per foot of anchor wall

P_P = resultant of the passive pressures in front of the anchor wall

P_A = resultant of the active pressures in back of the anchor wall

For homogeneous soils with S-case strengths

$$P_P = \gamma H^2 \, K_P/2 \tag{5-4}$$

and

$$P_A = \gamma H^2 \, K_A/2 \tag{5-5}$$

where K_P and K_A are passive and active earth pressure coefficients given in Equations 4-3 and 4-4 evaluated with the same effective angle of internal friction used for stability analysis of the retaining wall but with zero wall friction. For homogeneous soils with Q-case strength parameters, ($K_A = K_P = 1$)

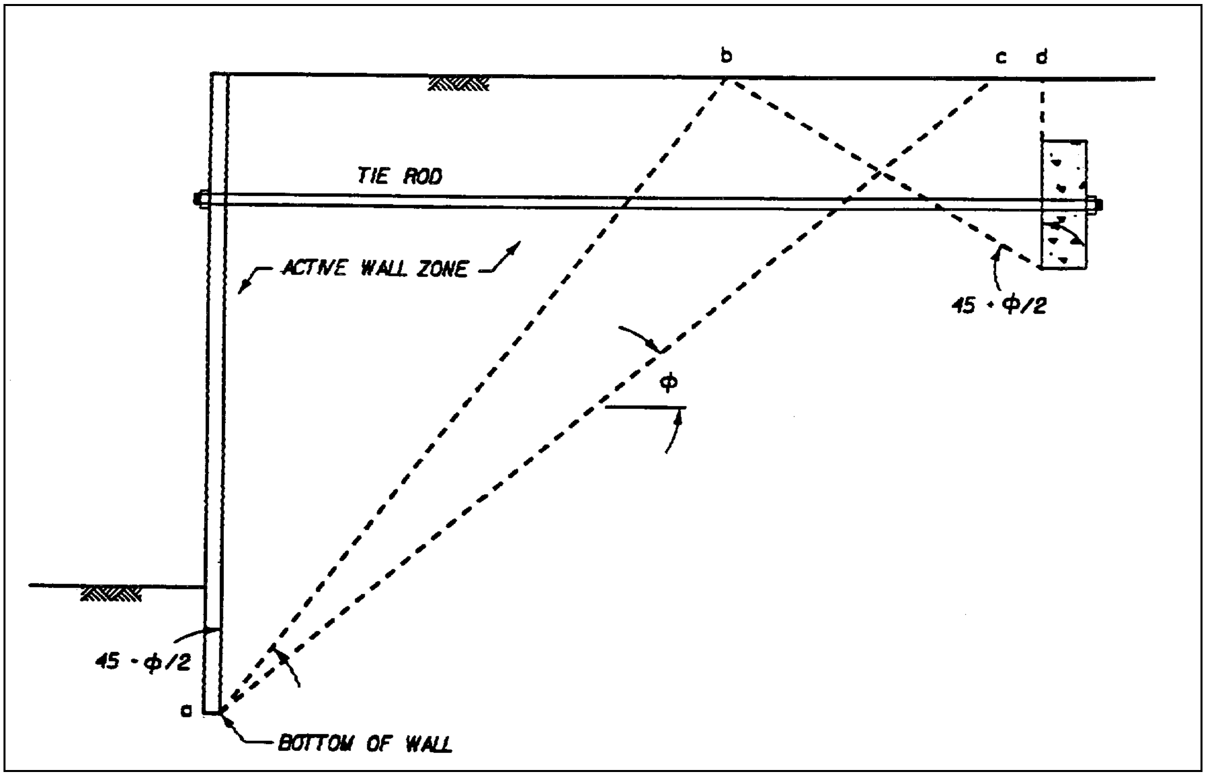

Figure 5-8. Minimum anchor - wall spacing for full passive anchor resistance in homogeneous soil

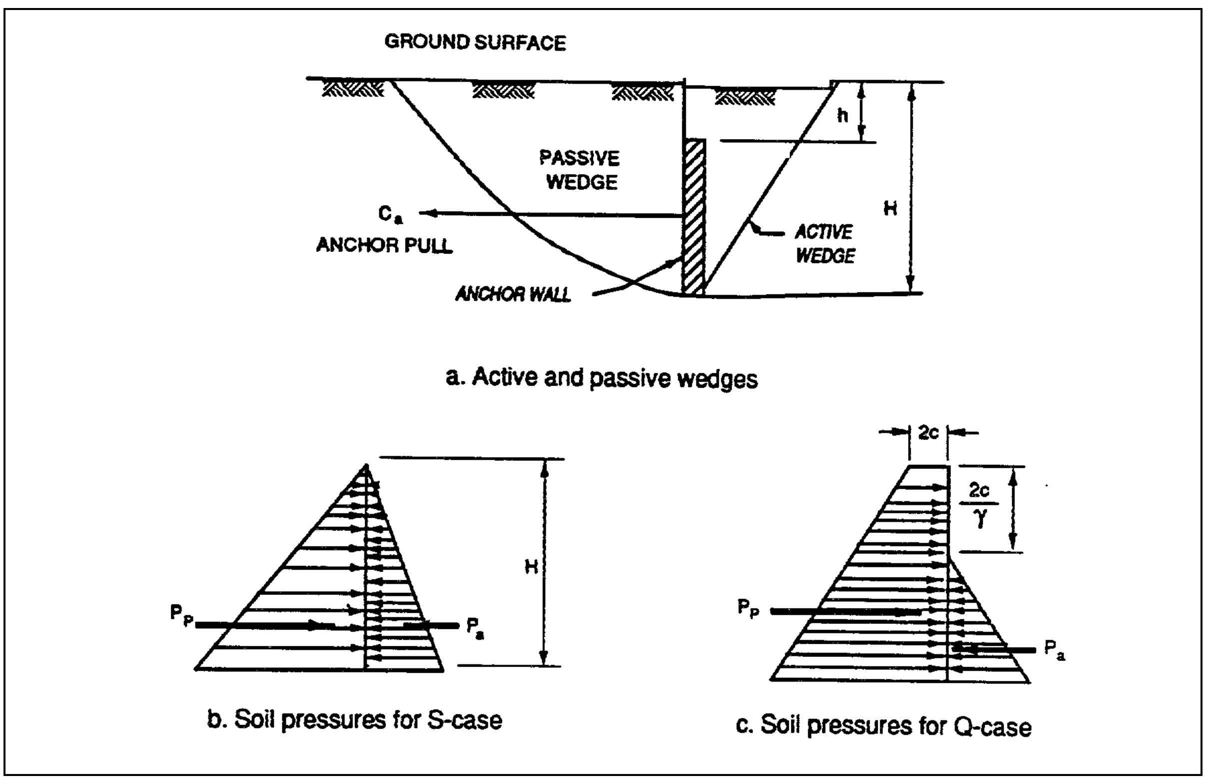

Figure 5-9. Resistance of continuous anchor wall

$$P_P = \frac{\gamma H^2}{2} + 2c_H \tag{5-6}$$

and

$$P_A = \frac{\gamma H^2}{2} - 2cH + \frac{2c^2}{\gamma} \tag{5-7}$$

where c is the effective soil cohesive strength used for stability analysis of the retaining wall.

(c) Reduced anchor wall capacity. When physical constraints require violation of the minimum spacing between anchor wall and retaining wall, the attendant reduced anchor wall capacity should be evaluated by the procedures discussed by Terzaghi (1934).

(d) Structural design of sheet pile and concrete anchor walls. Sheet pile anchor walls should be designed for maximum bending moment and shear under the stress limitations delineated in Chapter 6. Concrete anchors should be designed under the American Concrete Institute (ACI) 318 (1983) specifications for concrete structure in contact with the earth.

(2) Discontinuous anchors. Discontinuous anchors (or dead men) are usually composed of relatively short walls or blocks of concrete. The stress distribution ahead of a dead man is illustrated in Figure 5-10a and a free-body diagram is shown in Figure 5-10b. The capacity of a dead man near the ground surface for S-case strengths (c = 0) may be taken as

$$C_a = L(P_P - P_A) + (1/3)\ K_o\ \gamma \left(\sqrt{K_P} + \sqrt{K_A}\right) H^3 \tan(\phi) + W \tan(\phi) \tag{5-8}$$

and for Q-case strengths, (ϕ = 0)

$$C_a = L(P_P - P_A) + 2cH^2 + LBc \tag{5-9}$$

where

L = length of the dead man parallel to the retaining wall

B = thickness of the deadman perpendicular to the retaining wall

P_A and P_P = resultants of active and passive soil pressures (Equations 5-4 through 5-7), respectively

ϕ and c = effective (factored) angle of internal friction and cohesive strength, respectively

K_P and K_A = passive and active earth pressure coefficients evaluated for effective strengths (Equations 4.3 and 4.4)

K_o = at-rest pressure coefficient which may be taken as

$$K_o = 1 - \sin(\phi) \tag{5-10}$$

(3) Anchors at large depth. Capacities of anchors at large depth below the ground surface may be taken as the bearing capacity of a footing located at a depth equal to the midheight of the anchor (Terzaghi 1943).

(4) Grouted anchorage. Grouted anchorage consists of tie rods or tendons installed in cased, drilled holes with their remote ends grouted into competent soil or rock as illustrated in Figures 2-2c and 5. The grouted length must be fully outside the active wall zone (line ab in Figure 5-5). Tie rods must be designed to resist the anchor force determined from wall stability analysis plus any preload applied for alignment or limitation of initial deflections. The capacity of all grouted anchors, which should develop the yield strength of the tie rod, must be verified by proof tests by loading to 110 percent of their required resistance. At least two anchors should be subjected to performance tests by loading to 150 percent of their design capacity.

(5) Pile anchors. Capacities of anchors composed of tension piles or pile groups, Figure 2-2, should be evaluated by the procedures set forth in EM 1110-2-2906.

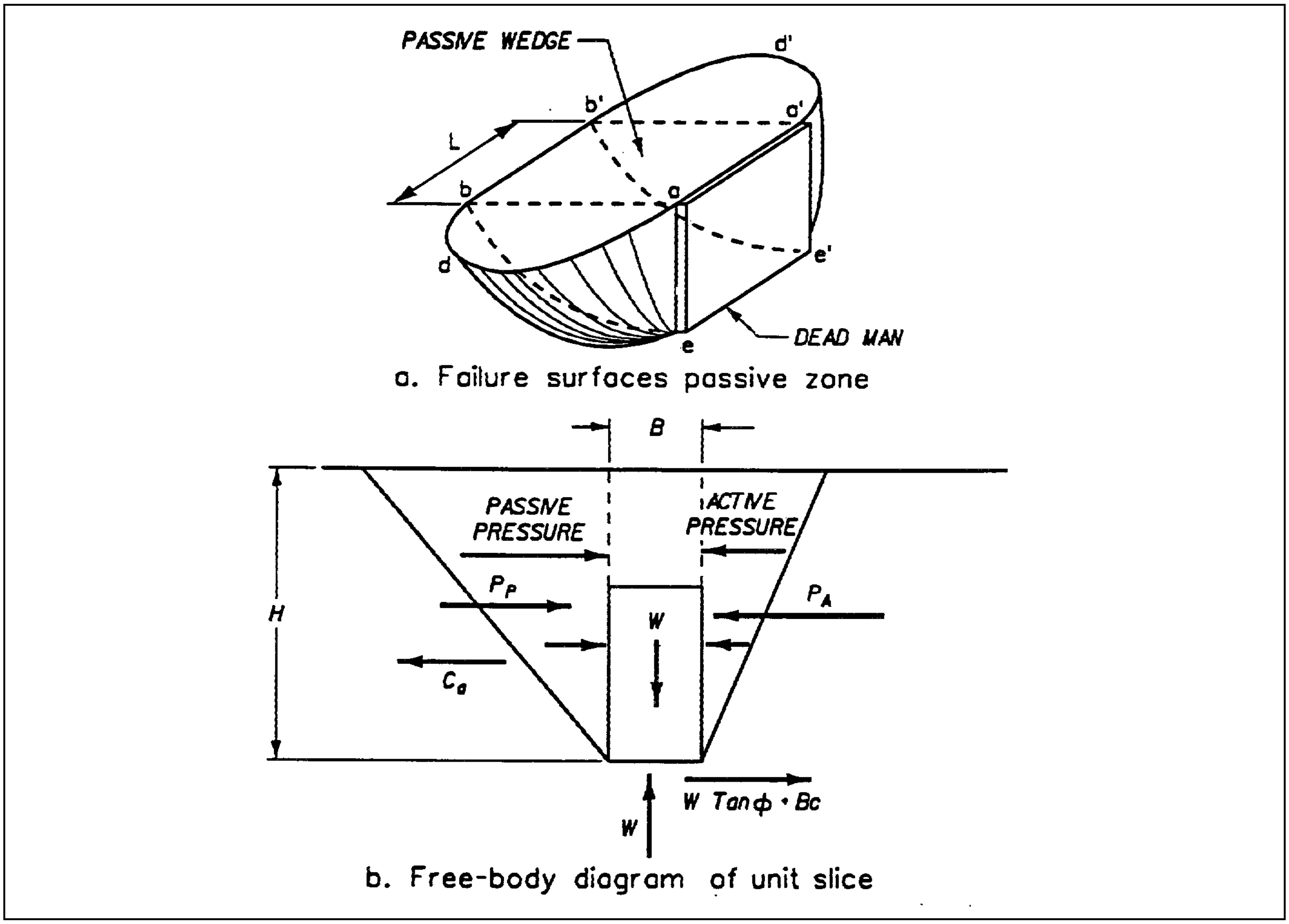

Figure 5-10. Resistance of discontinuous anchor (dead man)

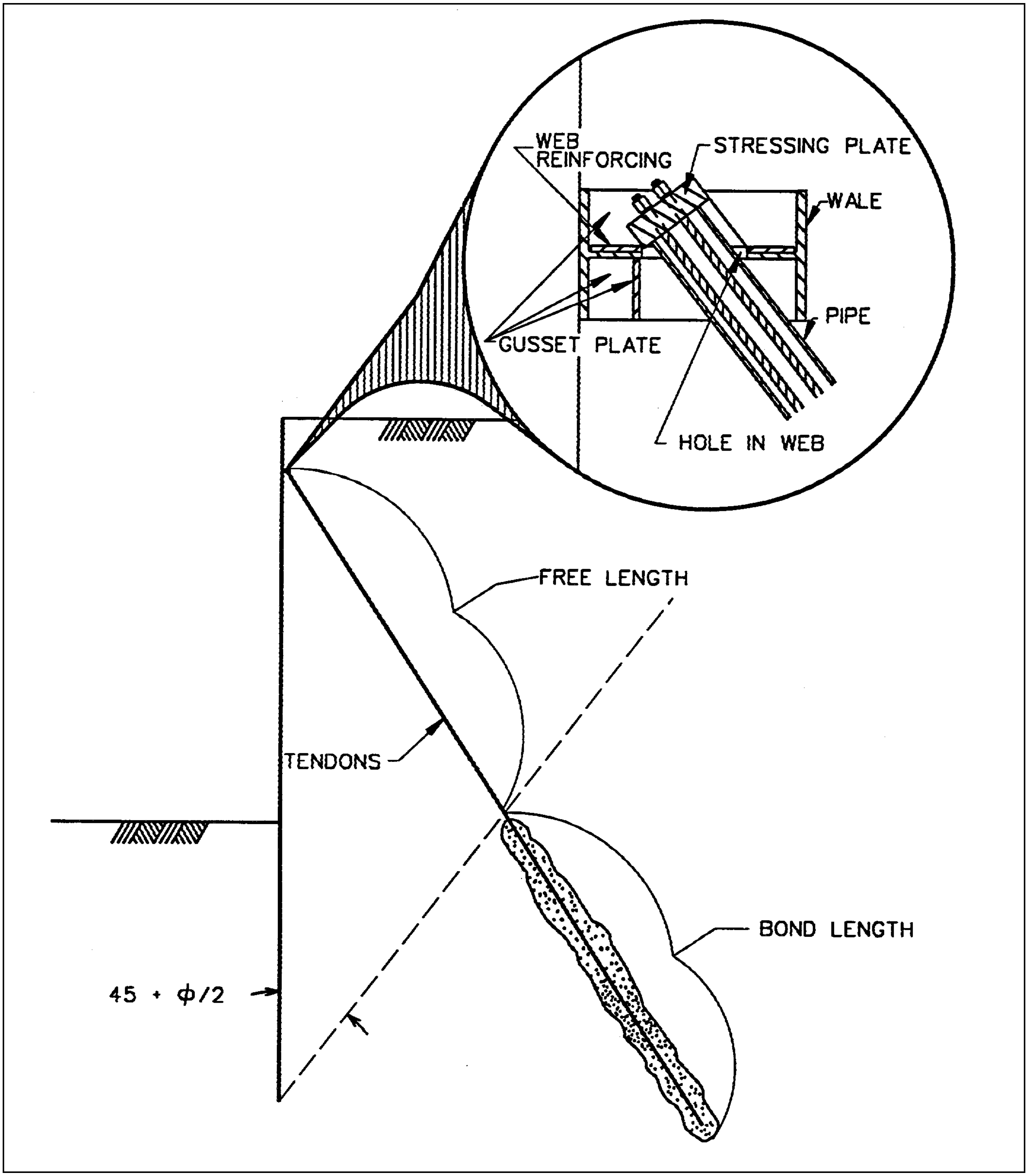

Figure 5-11. Grouted anchors